Caltrans

Earthquake Retrofit Guidelines for Bridges

Abstract

Memo 20-4 outlines the bridge retrofit procedure used by Caltrans as part of the Seismic Retrofit Program of California. This procedure contains four distinct phases: structural diagnostics, retrofit strategy development, elastic analysis bounding non-linear behavior, and retrofit design. Following Memo 20-4 are:

Attachment A:	STRUDL Modelling Guidelines
Attachment B:	Design/Detail Guidelines
Attachment C:	Special Considerations (seismic isolation; curvature analysis)
Attachment D:	Background and Ongoing Research Projects in Caltrans Retrofit Program

The primary philosophy for Caltrans retrofit program is to prevent collapse. The primary strategy to do this is to fully retrofit one bent (column/footing unit) per frame or bridge. However, the designer may demonstrate by analysis that collapse can be avoided without doing any retrofit. This type of "do nothing" strategy is an acceptable assessment. However, the designer must be cautioned to follow all load path demands and assure that no portion of the resisting structural frame is deficient. Seismic evaluation must not be limited to column or pier ductility capacities. It should be noted that serious localized damage could result from the philosophy to retrofit only to a capacity to prevent collapse. Closure and eventual replacement of many bridges following a serious earthquake should be expected as a result of the "prevent collapse" philosophy. Where structure serviceability is defined as a design requirement, a more conservative design approach than that outlined in this Memo 20-4 must be followed.

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Formerly titled, *Earthquake Retrofit Analysis for Single Column Bridges, updated March 1995*

Analysis and Design Overview

Structural evaluation at ultimate conditions (i.e., failure analysis) is an extreme challenge to an engineer. Cookbook or prefabricated processes do not lend themselves well to such a situation. Yielding of a single element in a particular mode may not cause collapse. A potential failure mechanism must be achieved before collapse can take place. The distribution, or redistribution, of additional load in a structural system after incremental yielding will be different for each structure. Therefore, each structure must be thoroughly evaluated. A flowchart is presented in Figure 1 which illustrates the bridge retrofit procedure recommended by Caltrans. The procedure includes four major tasks: structural diagnostics (steps #1-4), retrofit strategy development (step 5), elastic analysis bounding non-linear behavior (steps 6-11), and retrofit design (steps 12-16). This flowchart is meant to be an aid to the designer but in no way can it anticipate all possible variations. The basic task of the designer is to evaluate and retrofit the structure against all potential collapse modes.

Start

Review as-builts, site conditions (traffic, utilities), and obtain site seismicity data from Engineering Geology. Plan a site visit to verify as-built conditions.

Initial As-Built or Diagnostic Analysis

Designers first analyze retrofit candidate structures as if integrity is maintained and the structures respond linearly (i.e., all structural elements' strain levels remain in the linear-elastic range). This step should be performed even if there are apparent deficiencies so that a benchmark of member demands can be established. A conventional dynamic modal response spectrum analysis is performed and is indicated as step #2 in the flowchart (Figure 1). The purpose of this analysis is to evaluate the state of the structure under maximum credible earthquake loading. This analysis should be performed for both the tension and compression states. A proper analysis considers abutment springs and truss-like restrainer elements. Foundation springs are optional depending on subsurface condition. Caltrans *Bridge Design Aids* Chapter 14 (1) addresses abutment springs' evaluation with suggested procedures.

Uncracked column section properties shall be used when flexural moment ductilities are compared to tabulated allowable values shown in Figure 1. By modelling uncracked section properties, shorter periods are obtained. This results in higher force levels for typical bridge periods of magnitudes higher than the period corresponding to the peak response spectrum. If curvature analysis is considered in the bridge analysis, columns effective *EI* values as defined in Attachment A should be used to get a better estimate of displacement demands. These displacement demands are then compared to displacement capacities obtained using the curvature analysis approach.

Step #3 represents a check of the assumptions made in setting up the diagnostics analyses of step #2. Demand is compared to capacity. For example, if an abutment is assumed to possess a stiffness of 5000 kips/ft based on its initial stiffness, the backfill has a 500 kips ultimate capacity, and a dynamic analysis reports an abutment force of 1000 kips, then the analysis results are wrong due to the inappropriate stiffness of 5000 kips/ft assumed throughout the analysis. In reality, the columns will be forced to carry the load beyond the 500 kips load level at the abutments. Therefore, the abutment stiffness should be reduced iteratively.

Also, the existing hinge seats and restrainers must be analyzed. The six-inch hinge seats, common in box girder bridges in the '50's and '60's, have performed unsatisfactorily in past earthquakes. These hinge seats usually require seat extenders (2) in addition to cable restrainers. This restrainer and hinge seat assessment should be made prior to producing a dynamic analysis. Restrainer elongation must be small enough to prevent seat drop-off and restrainer forces must be small enough to prevent restrainer yielding or diaphragm failure. Diaphragms can fail if the restrainer tensile forces are greater than the superstructure's capacity to hold restrainers. Tests performed at the University of California at Los Angeles (3) on type C-1 hinge restrainers with seven cables failed in the diaphragm. The type C-1 standard was changed to a 5-cable unit based on the UCLA tests (Figure 2b). If the 7-cable restrainer system is present on a structure, modifications may be necessary to correct force levels or hinge seat travel using a pipe seat extender (Figure 2a). In addition, the designer must conduct a strength analysis of the existing diaphragm and connections to the superstructure. Older restraint systems cannot be assumed to be adequate and should be checked.

Results obtained from STRUDL analyses for design of restrainer units have proven to be inappropriate because of the demand to resist extremely large elastic column forces which are not actually attained. The equivalent static method (Chapter 14 in *Bridge Design Aids*) has been used successfully to design restrainer units across superstructure hinges and simple supports. This method is recommended in the design of the restrainer units. It assumes column pinning and hinging as determined based on conditions of the retrofitted structure. Column pinning occurs when plastic moment capacity is not sustained over the whole range of displacement ductility demands (strength loss may be initiated at ductility levels lower than the demand ductilities), or where an existing pin condition is present. Column plastic hinging occurs when plastic moment is sustained over the whole range of displacement ductility demands. In this latter case, fixity is maintained even though plastic rotations are present.

If the assumption checks of step #3 are not satisfied, the structure and/or diagnostics model must be modified in such a way that assumptions made in the STRUDL model, and/or equivalent static model for the restrainer analysis, are consistent with the analysis results. This modification is represented by step #3a in the flowchart. It is important to keep in perspective the expected, reasonable accuracies associated with this type of dynamic analysis. Generally, results within 20% after one iteration are satisfactory. Additional refinement of computer models is wasted effort considering that final elastic forces are modified by ductility ratios for design purposes. The above steps also represent a typical earthquake analysis performed in the design process of new construction at Caltrans. The following steps #4 through #15 represent additional investigative effort required for retrofit work.

Column Ductility

Past design practice and detailing has proven to be inadequate in regards to the amount of transverse reinforcement and the development length or lap splice of longitudinal bars. Therefore, allowable ductility demand ratios lower than current "Z" values are imposed on poorly confined compression members. These values are tabulated and shown in the box on Figure 1. Better detailed sections may be permitted larger values. The flexural moment demand ratio, μ_F, is defined for retrofit projects as the ratio of the sum of the earthquake moment reported by the response spectrum analysis plus dead load moment divided by the nominal moment determined by column section analysis.

$$\mu_F = \frac{M_{EQ} + M_D}{M_n}$$

where:

μ_F : Flexural moment ductility ratio

M_{EQ} : Unreduced seismic moment demand based on response spectrum analysis.

M_D : Dead Load moment.

For single-column bents where transverse loading direction governs the ductility demand, dead load moments can be considered equal to zero.

M_n : Nominal moment computed based on concrete compressive fiber strain equal to 0.003, and probable material strengths. Typically, aged concrete specified @ 3250 psi is considered to have a compressive strength of 5000 psi. Yield reinforcement should be based on mill certificate or tensile test results if those are available in the bridge archives. If not, a nominal strength of 1.1 times specified minimum yield strength should be assumed, resulting in 44 Ksi and 66 Ksi for grade 40 and grade 60 reinforcement respectively.

A plastic hinge should be assumed to form in any region where the ductility demand μ_F is 1.5 or greater. Any location where a plastic hinge is assumed to form should have continuous longitudinal reinforcement or have a shell enclosing lap splices of sufficient length (see Attachment B).

A plastic hinge will not occur at compression member ends unless proper bar development is available. If the column reinforcement development length or lap splice is not "reasonably" close to the required length, following guidelines stated in Attachment B, then the column connection should not be considered fixed in the model. Although a plastic hinge is expected to form where $\mu_F > 1.5$, higher values can be allowed without retrofitting as shown in box on Figure 1 when redundancy and ability to absorb energy in certain details are considered.

On multi-column bent bridges with larger amounts of redundancy such as several sets of three (or more) column bents, the larger number of maximum allowable ductility range (see Figure 1) may be used on columns.

On single-column bent bridges, the larger number should not predominate (more than 33% of the fixed column ends) the range of ductility demands for the total bridge.

Encasing columns in steel jackets, as shown in Figure 5, is the standard approach adopted by Caltrans to enhance column ductility. Meanwhile, high strength fiber composite wrap and vinyl-coated wire wrap have been successfully tested at UCSD.

The Caltrans current approach using modal analysis utilizes a comparison between demand forces and strength capacities of ductile members. However, a displacement check is needed when STRUDL CQC displacements exceed ⅙ of the diameter (round columns) or ⅙ the dimension (rectangular columns) in the direction of displacement (4). Under these conditions, computation of ultimate displacement of columns using curvature ductility analysis is recommended. This approach should be applied with a margin of safety that only the designer can prescribe since values of curvature at ultimate (Φu) are established based on an expected concrete strain failure or longitudinal steel strain beyond which slippage is initiated. A thorough description of this computational procedure can be found in references {5-12}.

Throughout the discussion for single- and multiple-column bent structures it is understood that many iterations might be needed to refine the STRUDL model and establish a retrofit solution prior to scheduling a strategy meeting.

Pier Walls Allowable Ductility

Based on recent U.C. Irvine tests, an allowable ductility, μ_F, equal to 4 is permitted for weak axis flexural ductility of pier walls without any required retrofit. Approval to apply this criteria depends on overall structural stability and must be granted in a strategy meeting.

The weak axis specimens tested in U.C. Irvine were 1:2 scale models 127-inches tall, 10-inches thick and 36-inches wide. Vertical reinforcement was No. 4 bars at 8.5-inch spacing or 0.56%. D7 wire was used for horizontal reinforcement at 7-inch spacing or 0.15% (13).

Column and Column/Footing Retrofits

A. *Multi-Columns Bents*

The general strategy is to retrofit one bent per frame. However, retrofit in multi-column bridges can often be limited to columns because of common pin connection to footings. Also, if the bent contains more than two columns, it may not always be necessary to retrofit all of the columns.

Footing retrofits shall be avoided on multi-column bridges by allowing pins at column bases as often as possible. Pins can be induced by allowing lap splices in main column bars to slip, or by allowing continuous main column bars to cause shear cracking in the footing.

If a pin is allowed to form at the bottom of a column, no column casing is required at the bottom of the column regardless of whether column reinforcement is continuous or lap-spliced at the column/ footing interface. However, this assumes that column shear demands are below allowable values. In addition, sufficient shear capacity across the footing interface must be provided to resist seismic shear forces.

In the case where column longitudinal reinforcement is continuous in the footing, the pin may form in the footing and axial load capacity must be maintained as described in the following paragraph.

The footing and piles within $0.5d_f$ (d_f = footing depth) of the column face should be able to support the vertical D.L., including seismic overturning axial load, in the event of footing break-up. Ultimate seismic pile capacity as specified by the Engineering Geology Section or Geotechnical Engineers should be used for this evaluation.

For evaluation of moment and shear ductility ratios in a multi-column bent, the following steps are recommended:

1. Determine Nominal and Plastic Moment Capacities M_n and M_p of columns ($M_p = 1.3\ M_n$). This can be done with "Yield" Program. Where flared columns exist, an evaluation of the flared-section capacity and ductility must be made.
2. Calculate Column shear force, V_u, by applying plastic moment values for each column at expected plastic hinge locations (see Figure 3). It is important to note that in some cases elastic column shear forces govern the analysis if column plastic shear forces are of larger magnitude.
3. Determine axial forces due to overturning based on axial stiffness of columns in each bent.
4. Recalculate nominal and plastic moments, M_n and M_p, based on axial dead load plus or minus axial forces due to overturning (shear forces being applied at the center of mass of superstructure, see Figure 4).
5. Recalculate Column shear V_u based on revised M_p.
6. Reiterate until you have reasonable convergence between applied shear at center of mass of superstructure and revised column shear forces.
7. Evaluate ductility demands based on revised M_p.
8. Compare revised column shear forces to allowable values. See Attachment B for more detailed discussion on allowable shear stress inside and outside plastic hinge region.
9. If column shear stress exceeds allowable shear stress outside plastic hinge region, full height grouted shell is used.

It is important to mention that multi-column retrofits will be allowed a preferred maximum ductility of 6.0 with isolated locations up to 8, subject to strategy panel approval.

In some cases, superstructure and/or bent cap retrofits may be required to assure fixity at the retrofit column end whose ductility demand exceeds 1.5. In order to assure column plastic hinging and avoid a collapse mechanism in the superstructure, the designer should ensure that 1.2 times the nominal moment strength of an effective width of superstructure is greater than the algebraic sum of all demands. The demands shall include the plastic hinging moment and shear of the column, superstructure gravity loads, prestress secondary moments, horizontal seismic loads, etc. This evaluation must be made in both the longitudinal and transverse directions. This requirement may be relaxed if a collapse condition is not present and approval obtained at a strategy meeting. Prestressing is an efficient option in enhancing cap beam moment capacity and improving beam/column shear transfer to help resist transverse seismic forces. With a post-tensioned cap beam, it might be only necessary to replace or widen the end regions of the cap beam. In these regions additional mild steel may be added, particularly to the positive moment steel. With all mild-steel design for cap beam retrofit, the cap beam probably needs to be widened, or replaced, over the full length. This is difficult because of the need to break through box girder webs, requiring superstructure support separate from cap beam support.

Vertical accelerations must be investigated to assure resistance against punching shear at all columns. If significant cracking in the superstructure, based on analysis, is assumed in the vicinity of the column, vertical acceleration demands could be catastrophic. Vertical demand should not be less than 1.5 gravity load. Stirrup reinforcement crossing through the assumed crack plane circling the column must be sufficient to resist the factored gravity load demand. The crack zone can be assumed at a distance 'd' (d = superstructure depth) from the face of the column. All girder and cap stirrups within that zone can be assumed effective against vertical demands.

Pinning the top of the column using an extra strong steel pipe drilled down the center of the column is an option that can be considered to reduce flexural and shear demand on the bent cap. However, in this latter case, column footing retrofit might be needed to ensure column stability. Alternate solutions allowing flexural hinging to occur in the bent cap should be the designer's last recourse provided sufficient rotational capacity exists. Consulting SASA/SEITECH or requesting direction from the strategy meeting panel on that issue is deemed to be quite important.

When checking superstructure plastic hinging in the longitudinal direction, use an effective width of superstructure to calculate the moment capacity (see Memo 20-6).

Generally, the total superstructure width would not be expected to contribute to the resisting strength because strains in the vicinity of the column would tend to be relatively large as compared to adjacent sections of superstructure.

When evaluating footing modifications, Engineering Geology or Geotechnical Engineers should be contacted for approximate ultimate pile and/or soil capacities. It is believed that, in some cases, piles under dynamic load possess ultimate compressive capacities at least four times their service load. The designer should take advantage of ultimate dynamic capacities, but must also realize that capacities may be greatly reduced by physical pile properties, reinforcement details, and connections. Diminished pile/soil friction, especially for end bearing piles, can greatly reduce or eliminate tensile capacity. End bearing piles in soft or saturated soils may have greatly reduced compressive capacity due to slenderness (l/r ratio) limitations. It should be noted that these ultimate capacities for retrofit designs are not to be used for new designs.

When tension capacity is needed, the use of standard tensile/compression piles are preferred to the use of tie-downs. In strong seismic events, large strain movements in footings are associated with tie-downs. Generally tie-downs cannot be prestressed to reduce strain movements without overloading existing piles in compression. The tie-down strains are probably not a serious problem with short columns where P-Δ effects are minimal. Also, tie-downs should be avoided where ground water could affect the quality of the installation. Soft cohesive soils (i.e., bay mud) pose an engineering problem for tie-downs or tensile piles. Several tensile pile type installations, including pre-loaded steel pipe pile/tie-down systems, are being tested on the Southern Freeway Viaduct as part of Caltrans' sponsored research on tension pile capacities. This pile/tie-down system would have the advantage of providing tension capacity without overloading existing piles in compression in addition to limiting footing rotational movement. Results will be available in late 1992. When a specific uplift resistance is required, tension piles should be identified on plans with a specified tip deeper than for compression piles. This issue has previously caused confusion to the contractor since desired tension values (ex., 50-ton piles) were designated as if they were compression piles. It is important for the designer to coordinate with specifications writers on this issue in order to convey that information to the contractor who is responsible for the driving operation.

B. Single Column Bent

A general rule of thumb is to fully retrofit one column (Class F Retrofit) per frame containing single column bents. If a column has been identified as one which is yielding, one of two options is available:

1. The column may be modified with a Class P retrofit (see Figure 5). These columns should be assumed to be pinned at their yield location in successive analyses. Keep in mind that the joint has some unaccounted reserve because the rubble will not be a frictionless pin. Note that the footing is not modified when a Class P retrofit is selected. Regardless of whether column reinforcement is continuous or lap-spliced at the column/footing interface a Class P retrofit is used where column is assumed to pin during the earthquake. If the column is identified as one which could fail in shear, a grouted full-height shell should be used. If a pin forms where column longitudinal reinforcement is continuous into the footing or at the bottom of a column that has a full-height shell, footing axial load capacity must be maintained as described in Section A.

2. The column may be modified with a Class F retrofit (see Figure 5). These columns should be assumed to remain fixed in successive analyses, keeping in mind that the column can hold at most its plastic moment and still possess a ductility capacity of about 4 to 6. Note that the footing usually requires modification when a Class F retrofit is selected. Figures 6 and 7 illustrate typical footing modifications designed to increase the footing and column/footing connections' moment holding capacity. The use of tie-downs to develop tension capacity in the footing should be avoided for tall single column bents. If a column is identified as one which could fail in shear, a grouted full-height shell should be used.

Option (1) is a relatively inexpensive alternative (costing a few thousand dollars per column) and should be the most frequently used option. It offers protection against total axial failure while allowing controlled flexural joint failure. Option (2) is a more expensive alternative (costing \$50,000 to \$100,000 per column) and should be selected prudently.

Single-column retrofits are permitted a preferred maximum ductility demand of 4.0, with increases up to 6.0 at isolated locations, subject to strategy panel approval.

When checking superstructure nominal moment capacity for single column bents against column plastic moment, only the longitudinal direction should be evaluated (see Memo 20-6).

In addition, it is important to note that the flexural ductility factor μ_F for single-column C-bents, whether retrofit or new, shall not exceed 2 in both orthogonal directions. A C-bent shall be defined as a single-column bent with the column located entirely outside the middle ⅓ width of the bent cap.

Retrofit Strategy

Steps #5 and #5a illustrate the selection process of the column retrofit strategy. It should be kept in mind that there are many satisfactory solution strategies and related assumptions. Therefore, to avoid confusion, both the designer and checker should be involved in retrofit strategy development. Experience in bridge response and nonlinear behavior is important at this step. Therefore, the designer, checker, and design senior should arrange strategy meetings with supervisory and specialty personnel to assist them in strategy development. The objectives of the strategy meetings are:

- offer seismic retrofit project engineers strategy support or alternative approaches
- determine that standard seismic retrofit details are being fully utilized and that aesthetics issues have been addressed
- alert specialty personnel of seismic retrofit problem areas where standards don't apply
- establish alternative acceptable procedures to satisfy retrofits when unusual problems are encountered (i.e., curvature ductility, outrigger strength, seismic isolation, soft foundation soils, etc.)
- recommend alternative analyses when low level ductility demands exist, displacements are physically limited, bridge site is in a low-risk seismic area, etc.
- inform project engineer of solutions to similar problems by other design sections
- keep supervisory personnel briefed on seismic retrofit details development
- achieve consensus agreement economical and practical retrofit strategies
- provide district personnel information for potential traffic control, right of way, utility, and environmental problems.

The designer and project engineer should be expected to have completed the diagnostics analysis, summarized the state of columns, restrainers/hinges and abutments, and have a proposed solution prior to scheduling a strategy meeting. The designer should be prepared to discuss solutions considered, and reasons for rejections and selections. Tables similar to the one shown in Figure 8 are recommended for strategy meetings. Seismic Retrofit General Plans employing an indexing system to identify location and type of retrofit work along a structure should be presented. For the strategy meeting, an existing as-built General Plan can be used to describe proposed retrofit measures. When reasonable, any foundation and column modifications should be indicated on the elevation view of the General Plan. Figure 9 illustrates these recommendations. General Plans of this type have proven extremely useful in strategy meetings. The benefit of having a retrofit legend on the G.P. is that future reviewers will be able to scan a seismic retrofit G.P. and know where retrofit modifications were made. Seismic retrofit General Plans are kept in the SEITECH Section and are available for reference.

It is no longer necessary to present all bridge retrofit strategies at a formal strategy meeting. If the designer and section leader are comfortable with the retrofit solution, the meeting may be omitted. However, the designer is responsible for interacting with District or Sacramento Design personnel to resolve roadway issues, and submitting a memo documenting pertinent strategy information. The retrofit strategy memo should include, as a minimum, the following items:

1. the strategy selected and supporting reasons,
2. the alternative schemes considered and reasons for rejection,
3. direction received from SASA, SEITECH or the Strategy Meeting participants,
4. roadway issues, (i.e., traffic, right-of-way, utilities, environmental, leased space, etc.) which contributed to retrofit decisions,
5. geotechnical and foundation allowables and restrictions,
6. any other data which supports the reasons for selecting or rejecting schemes,
7. a tabular summary of engineering data (i.e., tension/ compression model column moment ductility demands for the as-built and retrofit conditions, shear capacity/demand comparisons, assumed concrete strength(s), rebar grade(s), pile/soil support allowables, ARS curve and depth of alluvium used, assessment of superstructure capacity/ demand both transversely and longitudinally, risk rating on the bridge retrofit list, etc.), and
8. appropriate cost data if relative to strategy decisions.

Each bridge in a project should be summarized separately.

The project designer and section leader must concur on the content of this memo. The memo should give a complete summary of the strategy decision process to someone unfamiliar with the seismic vulnerability of the structure. A copy of the G. P., showing intended retrofit work descriptions (legends) and locations should be attached to the memo. The memo should be addressed to the Design A or B supervisor with copies to SASA and SEITECH, and signed by the section leader.

The section leader must be advised of and approve selected strategies, whether a meeting will be requested or omitted. For difficult situations, the designer is encouraged to seek comments/assistance from SASA/SEITECH before settling on a strategy. Specific SEITECH personnel have been assigned to Design Sections and External Finance seismic reviewers. Traffic and environmental concerns may require modification of strategies. It is important to interact with District/Sacramento Design personnel to arrive at mutually satisfactory details. Those factors may be cause to delay projects, but should not be cause for compromising the effectiveness of the retrofit. The OSD project engineer is required to keep District/Sacramento Design personnel fully informed of project progress and details. In addition, the project engineer needs to determine whether additional work is scheduled for the subject bridge or whether it is scheduled for replacement by Structures Maintenance (deficient) or District (new alignment or widening). The decision of whether to retrofit or wait for replacement rests with the District. However, a recommendation may be made at the strategy meeting and elevated to the Office Chief if necessary.

A type selection meeting may be scheduled regarding the subject bridge in case of widening or rehabilitation if requested by one of the design supervisors. Regardless of whether a type selection meeting is held, a type selection memo should be produced and distributed. If the meeting is not held, a copy of the memo and a G.P. should be distributed to those who would normally attend the type selection meeting, i.e., Construction, Maintenance, Aesthetics, Specifications, District, and Geology.

In summary, the designer's goal is to determine an economical retrofit strategy in which load paths are traced and capacities are found to be sufficient to maintain the integrity of the structure. The selected strategy will determine the fixity conditions used in supplemental analyses. The typical box girder bridge can be considered relatively forgiving. If a reasonable load path is provided to transmit the seismic loads to the ground, the load carrying system within the structure will find it and make use of it. A typical first strategy might be to identify column retrofits (Class P or Class F casings, full or partial length steel shell, fiber epoxy shell). It is also important to provide adequate restrainers at all hinges to provide a path through the superstructure to allow redistribution to adjacent frames, columns and abutments. To accomplish this goal, additional restrainers may be required even if the subject bridge had been retrofitted in the Phase I Retrofit Program. Possible restrainer work might include adding restrainers to increase strength, adding abutment tie-backs (see Figure 10), lengthening restrainers to reduce stiffness, and/or increasing effective seat width with pipe seat extenders (see Figure 2a). Possible footing modifications might include adding piles and/or increasing the size of the footing, adding tension tie-downs, or adding a top mat of steel with concrete cover (see Figures 6 and 7). Superstructures may need strengthening, column fixed connections at ends may need improvement, outriggers may need replacement, restrainer anchorages may need reinforcing, and other unusual details may be required in extreme cases.

Retrofit Model Analysis

Tension and Compression Models

After the retrofit strategy has been determined, an elastic analysis of a more refined model of the subject bridge is performed. This analysis is run iteratively in an attempt to bound strength and displacement demands on the structure due to earthquake loadings. Steps #5 through #11 illustrate the recommended procedure for seismic retrofit projects.

In Step #6 two dynamic models are used to bound the assumed nonlinear response of the bridge; a "tension model" and a "compression model". Two models are used because the bridge possesses different characteristics in tension versus compression. As the bridge opens up at it joints, it pulls on the restrainers. In contrast, as the bridge closes up at its joints, its superstructure elements go into compression.

In the tension model, the superstructure joint elements, including the abutment, are released longitudinally with the truss restrainer elements connecting them at the joints (see Figure A4, Attachment A). In the compression model, all of the restrainer elements are inactivated and the superstructure elements are locked longitudinally to capture the structural response in modes in which the superstructure tends to close up and go into compression, mobilizing the abutments when applicable.

Using the peak abutment force and the effective area of the mobilized soil wedge, the peak soil pressure is compared to a maximum abutment capacity of 7.7 Ksf and lateral pile capacity of 40 Kips per pile. If the peak soil pressure exceeds the soil capacity, the analysis should be repeated with a reduced abutment stiffness. It is important to note that the 7.7 Ksf bearing pressure is based on a reliable minimum wall height of 8 feet. If the wall height is less than 8 feet, or if the wall is expected to shear off at a depth below the roadway less than 8 feet, the allowable passive soil pressure must be reduced. The allowable pressure is reduced in these cases by multiplying 7.7 Ksf times the ratio of $(L/8)^2$, "L" being the effective height of wall. Furthermore, the abutment wall diaphragm (structural member mobilizing soil wedge) shear capacity should be compared to the demand force. Abutment spring displacement is then evaluated against the acceptable level of displacement. This deflection will vary depending on the gap between the superstructure and backwall for seat abutment, or whether a diaphragm abutment exists. However, a net displacement of about 0.2 ft. at abutments should not be exceeded (net displacement 0.2 ft. does not include the gap displacement or soil mobilization displacement). Field inspections after the 1971 San Fernando Earthquake suggest that abutments which moved up to 0.2 ft. in the longitudinal direction into the backfill soil appeared to survive with little need for repair. Abutments in which the backwall breaks off before other abutment damage occurs can be permitted to undergo much larger displacements. Larger displacements may also be satisfactory if a reasonable load path can be provided to adjacent bents and no collapse potential is indicated.

The seismic anchor slab or "waffle slab" could be used in a bridge retrofit strategy where the designer wishes to substantially stiffen the abutments (see Figure 11 and 12). This detail would attract larger seismic forces to the abutments and could reduce the amount of column, footing, or other retrofit which may be required in adjacent bents. The seismic anchor slab is more effective on shorter bridges with no hinges (see Sullivan Ave. OC, Bridge #35-186K and other structures in Earthquake Retrofit Project No. 40 on Route 280 in San Mateo), however, it has been proposed for use on larger structures with expansion hinges (by Imbsen and Associates for L.A. County). Several design issues regarding the seismic anchor slab are included in Attachment B.

In cases where it is not practical to restrain the superstructure longitudinally at an abutment, supplemental seat supports can be provided to prevent the superstructure from dropping.

For seismic loads in the transverse direction, the same general principles discussed above still apply. Wingwalls are tied to the abutment to stiffen the bridge transversely (see Figure 10). Spring stiffness calculations are shown in *Bridge Design Aids* 14-3. Other methods of stiffening abutments include the addition of large diameter cast-in-drilled hole piles on both sides of the abutment (see Figure 13). A good example of the latter approach is Burnt Mill Canyon Bridge (#54-859) on Route 138 in San Bernardino County. Most existing wingwalls provide little lateral support on the outside because the soil impact is small and the soil usually slopes away from the wall resulting in slight soil resistance. The 0.2 ft. displacement limit also applies in the transverse direction if the abutment stiffness is expected to be maintained. Larger deflections may be satisfactory if a reasonable load path can be provided to adjacent bents and no collapse potential is indicated.

Typically 4-foot diameter pile shafts can be added to abutments to resist large earthquake loads. For these shafts to be effective, abutments displacements should match pile shaft displacement capacity needed to mobilize the soil lateral capacity. Transverse resistance is offered through monolithically connected shafts on either side of the original abutment. Longitudinal tensile resistance is typically offered through shafts placed behind the backwall and then connected to the bridge superstructure with high strength rods through the backwall.

It should be remembered that in some cases, such as in highly curved bridges, abutments offer little help in reducing demands in a compression model or for transverse direction movement across the embankments (see Attachment A).

The designer should iterate through steps #7-11 until the dynamic analysis is producing results that are consistent with the retrofit strategy. It is not necessary to over-refine the analysis; 20% accuracy is sufficient considering that the design is performed based on ductility factors and not on elastic forces.

Estimate and Complete P. S. & E.

Structural plans and details must provide enough information that would enable the contractor to have a good estimate of quantities and construction procedures involved at the bidding stage. Dimensions should be clearly identified in order to show amount of concrete removal, available headroom and anticipated excavation [check with SEITECH (Ralph) for typical sheets on excavation and backfill limits, no standard sheet number available yet, see Figure 14].

ALLOWABLE MAXIMUM DUCTILITY DEMANDS μ_F Max ON POORLY CONFINED COMPRESSION MEMBERS		
COMPRESSION MEMBER TYPE	SINGLE-COLUMN BENTS	MULTIPLE-COLUMN BENTS
Round Columns	1.5 - 2.0	2.0 - 3.0
Rectangular Columns	1.0	1.5 - 2.0
Round Pile Extensions or Round Shaft (@ Bottom in soil only)	2.0 - 3.0	3.0 - 4.0

Note:
Allowable demands for pier walls should be assessed in Strategy meetings. Generally, pier wall weak axis demands may approach 4.0.

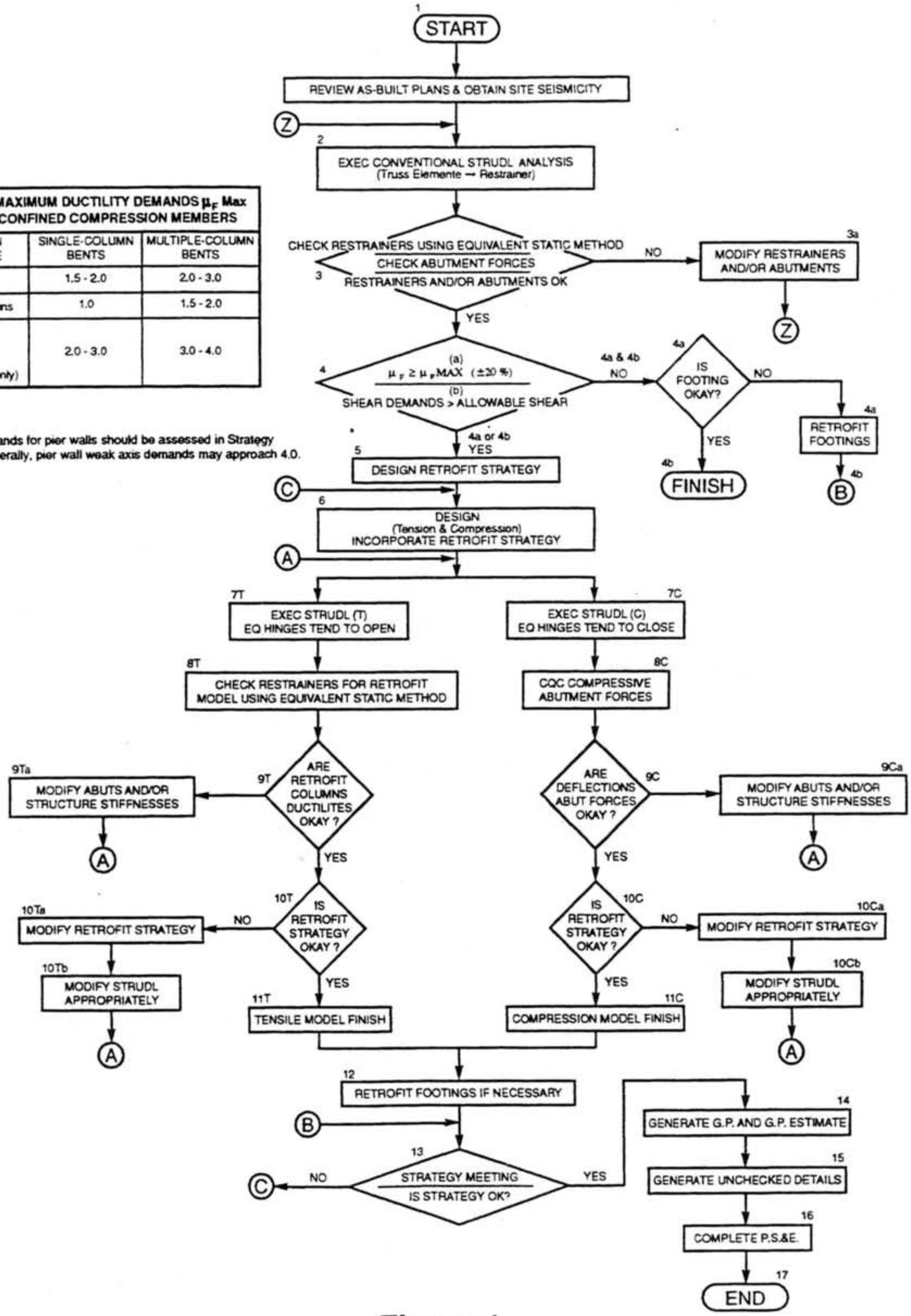


Figure 1

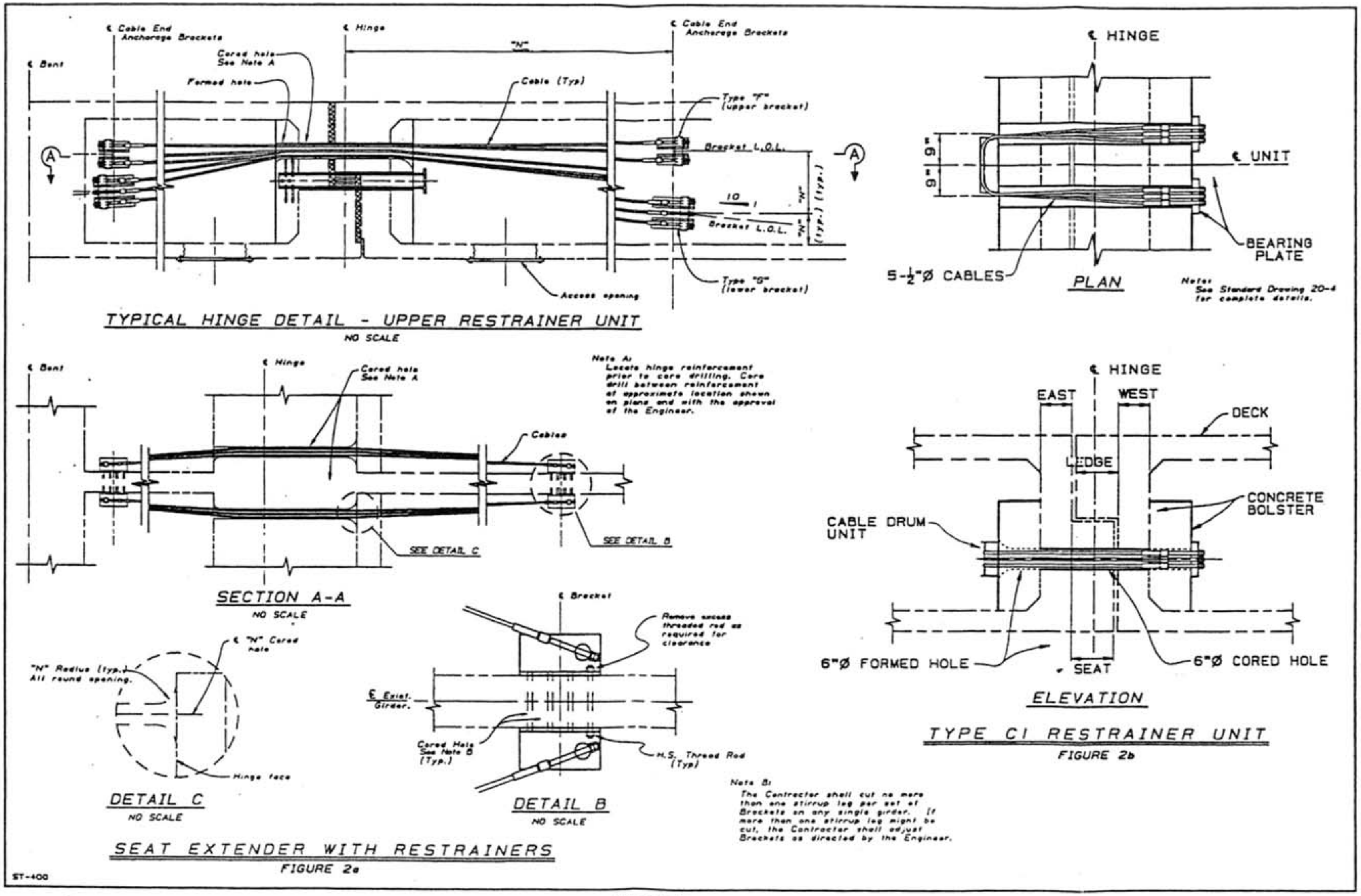
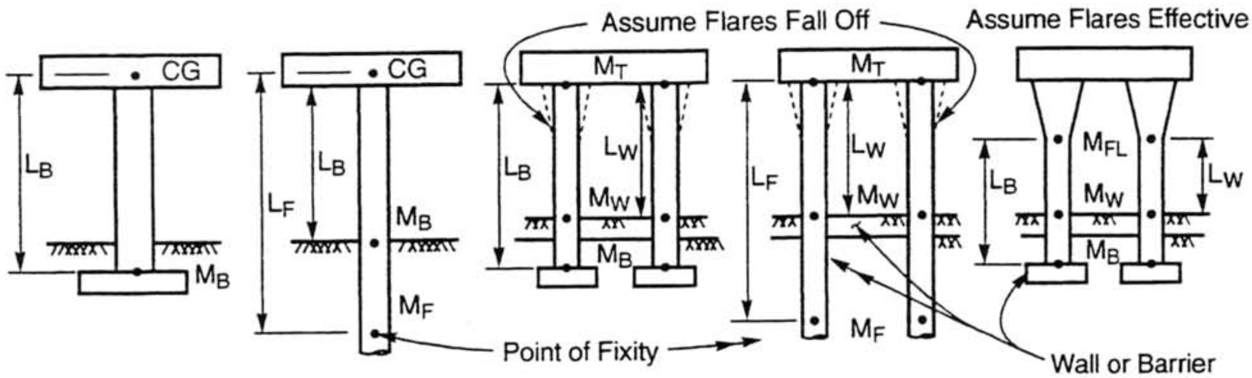

TYPICAL HINGE DETAIL - UPPER RESTRAINER UNIT
NO SCALE
SECTION A-A
NO SCALE
DETAIL C
NO SCALE
DETAIL B
NO SCALE
SEAT EXTENDER WITH RESTRAINERS
FIGURE 2a
Cable End Anchorage Brackets
Hinge
Bent
Cored hole See Note A
Formed hole
Cable (Typ)
Type "F" (upper bracket)
Bracket L.O.L.
Type "G" (lower bracket)
Access opening
Cables
SEE DETAIL C
SEE DETAIL B
Note A: Locate hinge reinforcement prior to core drilling. Core drill between reinforcement at approximate location shown on plans and with the approval of the Engineer.
"N" Radius (typ.) All round opening.
"N" Cored hole
Hinge face
Bracket
Remove excess threaded rod as required for clearance
Exist. Girder.
Cored Hole See Note B (Typ.)
H.S. Thread Rod (Typ)
Note B: The Contractor shall cut no more than one stirrup leg per set of Brackets on any single girder. If more than one stirrup leg might be cut, the Contractor shall adjust Brackets as directed by the Engineer.
ST-400
HINGE
UNIT
9"
9"
BEARING PLATE
5-½"Ø CABLES
PLAN
Note: See Standard Drawing 20-4 for complete details.
EAST
WEST
DECK
LEDGE
CONCRETE BOLSTER
CABLE DRUM UNIT
6"Ø FORMED HOLE
SEAT
6"Ø CORED HOLE
ELEVATION
TYPE CI RESTRAINER UNIT
FIGURE 2b

Figure 2

Transverse



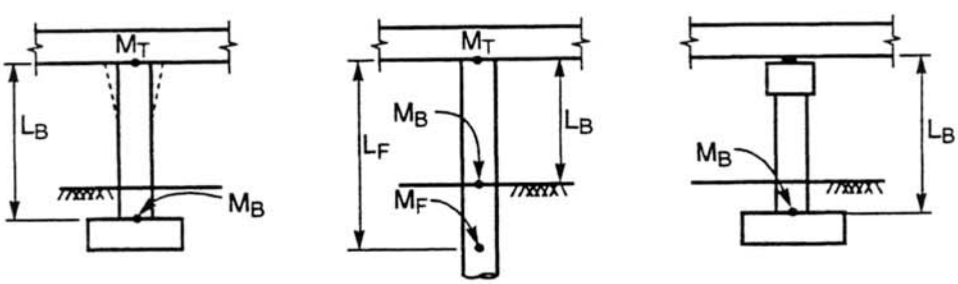
$$V = \frac{M_B}{L_B} \qquad V = \frac{M_B}{L_B} \text{ or } \frac{M_F}{L_B} \qquad V = \frac{M_T + M_W}{L_W} \qquad V = \frac{M_T + M_W}{L_W} \qquad V = \frac{M_{FL} + M_W}{L_W}$$

or

$$= \frac{M_T + M_B}{L_B} \qquad = \frac{M_T + M_F}{L_F} \qquad = \frac{M_{FL} + M_B}{L_B}$$

NOTE: 'L' could terminate at intermediate point on flare depending on ratio of applied moment to flare capacity, both of which vary with column height.

Longitudinal

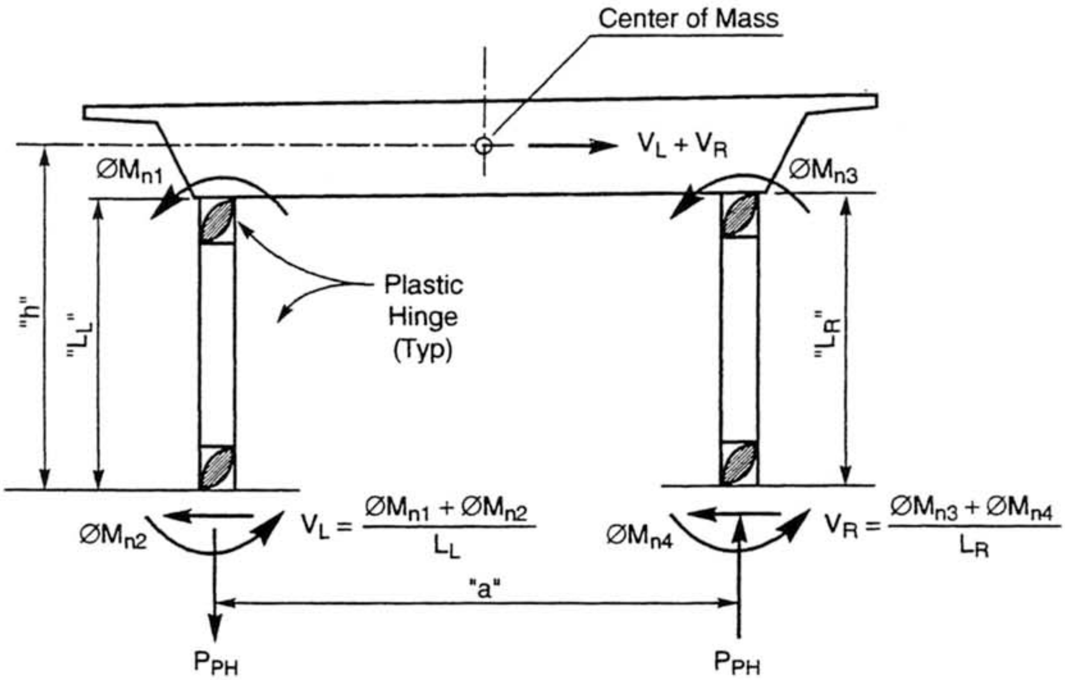


$$V = \frac{M_T + M_B}{L_B} \qquad V = \frac{M_T + M_B}{L_B} \text{ or } \frac{M_T + M_F}{L_F} \qquad V = \frac{M_B}{L_B}$$

NOTE: 'L' could be different in longitudinal and transverse directions due to restrictions such as retaining walls in direction of bent, etc.

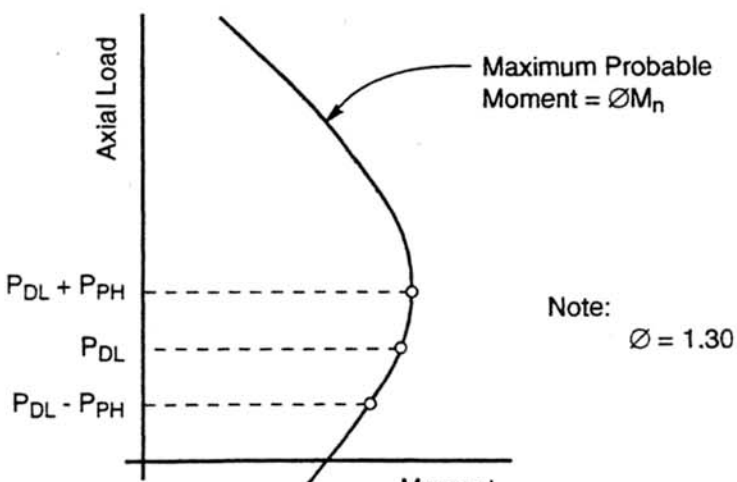
Calculation of Shear Force for Different Plastic Hinge Locations

Figure 3



$$P_{PH} = \pm \frac{(V_L + VR)h - \varnothing Mn2 - \varnothing Mn4}{a}$$

Axial Forces Due to Plastic Hinging



Column Interaction Diagram

Figure 4

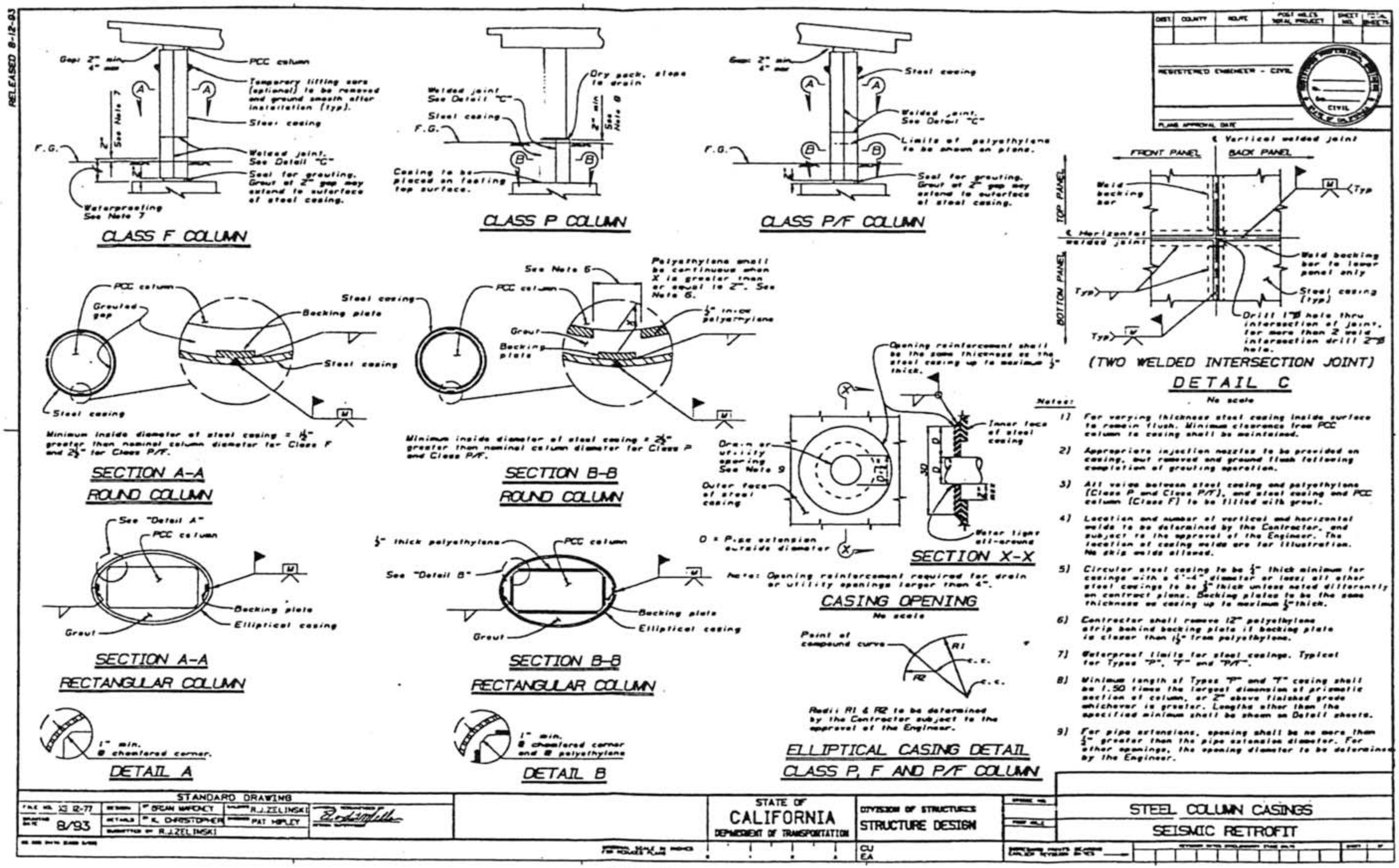

RELEASED 8-12-93
CLASS F COLUMN
CLASS P COLUMN
CLASS P/F COLUMN
PCC column
Steel casing
Welded joint. See Detail "C"
Seal for grouting. Grout at 2" gap may extend to outerface of steel casing.
Dry pack, slope to drain
Casing to be placed on footing top surface.
Limits of polyethylene to be shown on plans.
Waterproofing See Note 7
F.G.
SECTION A-A
ROUND COLUMN
SECTION B-B
ROUND COLUMN
Minimum inside diameter of steel casing = 1½" greater than nominal column diameter for Class F and 2½" for Class P/F.
Minimum inside diameter of steel casing = 2½" greater than nominal column diameter for Class P and Class P/F.
Grouted gap
Backing plate
Grout
See Note 6
½" thick polyethylene
SECTION A-A
RECTANGULAR COLUMN
SECTION B-B
RECTANGULAR COLUMN
Elliptical casing
DETAIL A
DETAIL B
1" min. @ chamfered corner.
1" min. @ chamfered corner and @ polyethylene
Opening reinforcement shall be the same thickness as the steel casing up to maximum ½" thick.
Inner face of steel casing
Outer face of steel casing
Water tight all-around
Drain or utility opening See Note 9
D = Pipe extension outside diameter
SECTION X-X
Note: Opening reinforcement required for drain or utility openings larger than 4".
CASING OPENING
No scale
Point of compound curve
Radii R1 & R2 to be determined by the Contractor subject to the approval of the Engineer.
ELLIPTICAL CASING DETAIL
CLASS P, F AND P/F COLUMN
FRONT PANEL
BACK PANEL
TOP PANEL
BOTTOM PANEL
Vertical welded joint
Horizontal welded joint
Weld backing bar
Weld backing bar to lower panel only
Drill 1" hole thru intersection of joint. for more than 2 weld intersection drill 2 hole.
(TWO WELDED INTERSECTION JOINT)
DETAIL C
No scale
Notes:
1) For varying thickness steel casing inside surface to remain flush. Minimum clearance from PCC column to casing shall be maintained.
2) Appropriate injection nozzles to be provided on casing, but removed and ground flush following completion of grouting operation.
3) All voids between steel casing and polyethylene (Class P and Class P/F), and steel casing and PCC column (Class F) to be filled with grout.
4) Location and number of vertical and horizontal welds to be determined by the Contractor, and subject to the approval of the Engineer. The location of casing welds are for illustration. No skip welds allowed.
5) Circular steel casing to be ⅜" thick minimum for casings with a 4'-4" diameter or less; all other steel casings to be ½" thick unless noted differently on contract plans. Backing plates to be the same thickness as casing up to maximum ½" thick.
6) Contractor shall remove 12" polyethylene strip behind backing plate if backing plate is closer than 1½" from polyethylene.
7) Waterproof limits for steel casings. Typical for Types "P", "F" and "P/F".
8) Minimum length of Types "P" and "F" casing shall be 1.50 times the largest dimension of prismatic section of column, or 2" above finished grade whichever is greater. Lengths other than the specified minimum shall be shown on Detail sheets.
9) For pipe extensions, opening shall be no more than ½" greater than the pipe extension diameter. For other openings, the opening diameter to be determined by the Engineer.
REGISTERED ENGINEER - CIVIL
STANDARD DRAWING
BRIAN MARONEY
R. J. ZELINSKI
K. CHRISTOPHER
PAT HIPLEY
8/93
STATE OF CALIFORNIA
DEPARTMENT OF TRANSPORTATION
DIVISION OF STRUCTURES
STRUCTURE DESIGN
STEEL COLUMN CASINGS
SEISMIC RETROFIT

Figure 5

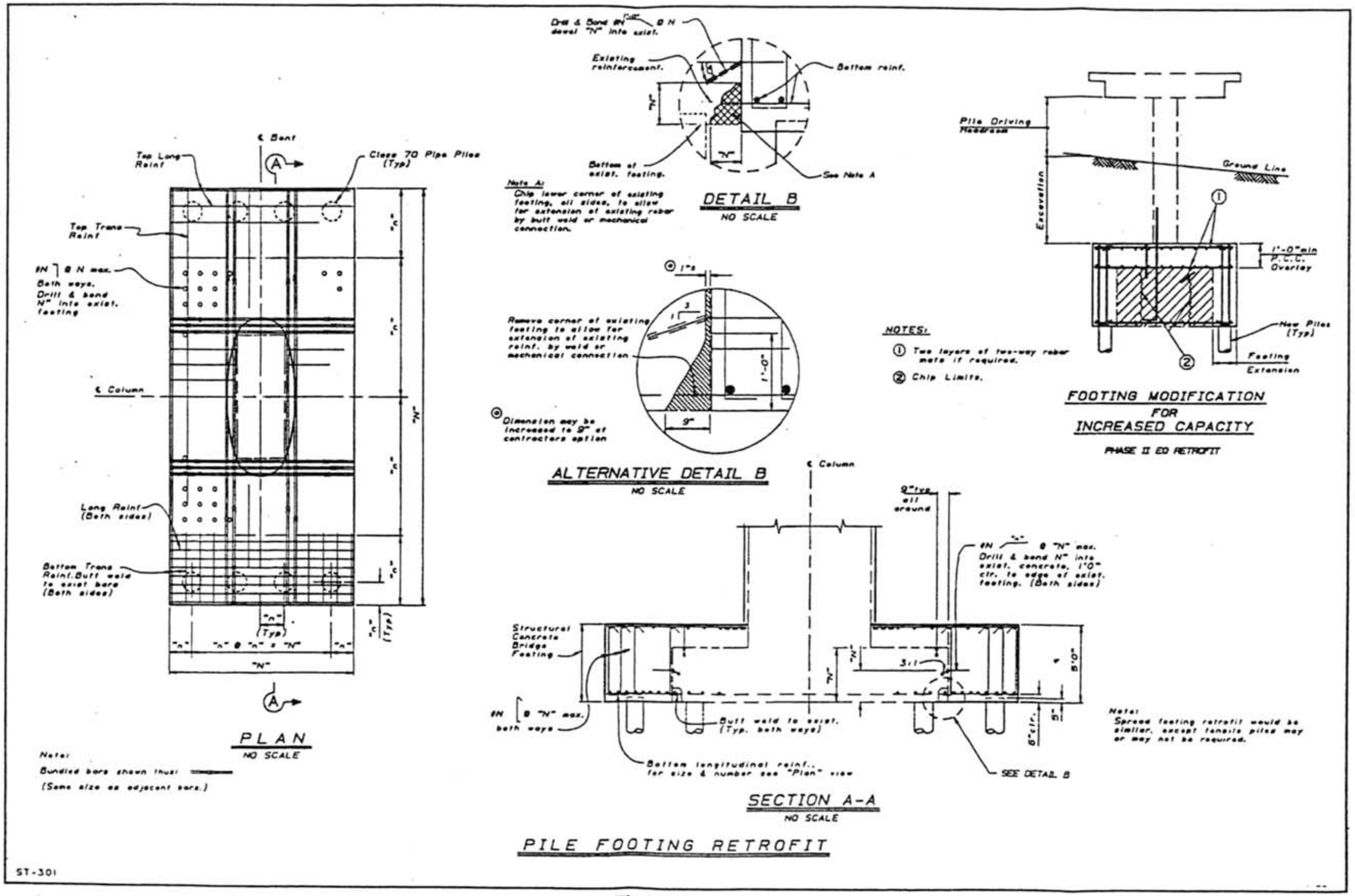

PLAN
NO SCALE
DETAIL B
NO SCALE
ALTERNATIVE DETAIL B
NO SCALE
SECTION A-A
NO SCALE
FOOTING MODIFICATION
FOR
INCREASED CAPACITY
PHASE II ED RETROFIT
NOTES:
① Two layers of two-way rebar mats if required.
② Chip Limits.
Note:
Spread footing retrofit would be similar, except tensile piles may or may not be required.
PILE FOOTING RETROFIT
ST-301

Figure 6

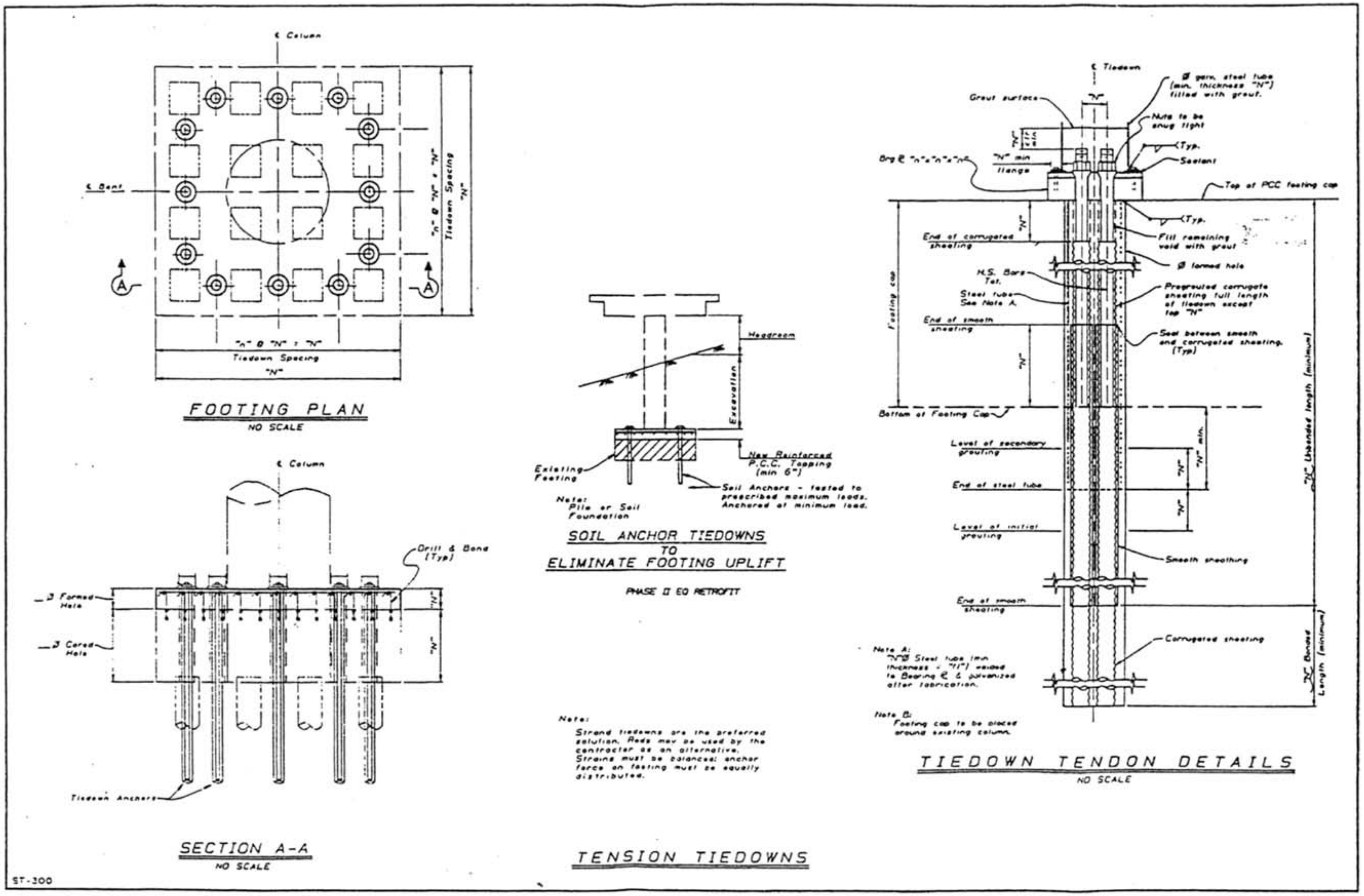

FOOTING PLAN
NO SCALE
SECTION A-A
NO SCALE
SOIL ANCHOR TIEDOWNS
TO
ELIMINATE FOOTING UPLIFT
PHASE II EQ RETROFIT
Existing Footing
Note: Pile or Soil Foundation
Headroom
Excavation
New Reinforced P.C.C. Topping (min 6")
Soil Anchors - tested to prescribed maximum loads. Anchored at minimum load.
Note:
Strand tiedowns are the preferred solution. Rods may be used by the contractor as an alternative. Strains must be balanced; anchor force on footing must be equally distributed.
TENSION TIEDOWNS
TIEDOWN TENDON DETAILS
NO SCALE
Grout surface
Top of PCC footing cap
Sealant
Nuts to be snug tight
Fill remaining void with grout
Ø formed hole
End of corrugated sheathing
End of smooth sheathing
Bottom of Footing Cap
Level of secondary grouting
End of steel tube
Level of initial grouting
Smooth sheathing
Corrugated sheathing
Footing cap
Drill & Bond (Typ)
Tiedown Anchors
ST-300

Figure 7

References

1. "Bridge Design Aids," California Department of Transportation.

2. "Cypress St./I-880 Hinge Restrainer Test," Office of Transportation Materials and Research, March 21, 1990.

3. Selna, L. and Malvar, J., "Full Scale Experimental Testing of Retrofit Devices Used for Reinforced Concrete Bridges," Report No. UCLA/EQSE-87/01, June 1987.

4. Mahin, S., and Boroschek, R., "Influence of Geometric Nonlinearities on the Seismic Response and Design of Bridge Structures," report to Caltrans Division of Structures, October 1991.

5. Park, P., and Paulay, T., "Reinforced Concrete Structures," John Wiley and Sons, Inc., New York, N.Y. 1975.

6. Priestley, N., and Seible, F., "Seismic Assessment and Retrofit of Bridges," Report No. SSRP-91/03, University of California at San Diego, La Jolla, California, July 1991.

7. "Seismic Behavior of Single Column Bents in Transverse Direction," Mark Seyed, Special Analysis Section, Caltrans, 1/30/92.

8. "Seismic Design Criteria for CYPRESS St. Viaduct," Mark Seyed, Special Analysis Section, Caltrans, 10/31/91.

9. "Foundation Stiffness Issues for CYPRESS St. Viaduct," Mark Seyed, Special Analysis Section, Caltrans, 12/16/91.

10. "User's Manual for COLx, Column Ductility Program," Mark Seyed, Special Analysis Section, Caltrans, 1/24/92.

11. "User's Manual for BEAMx, Beam Ductility Program," Mark Seyed, Special Analysis Section, Caltrans, 1/27/92.

12. "User's Manual for FRAMEx, Frame Ductility Program," Mark Seyed, Special Analysis Section, Caltrans, 2/10/92.

13. Haroun, M.A., Pardoen, G.C., and Shepherd, R., "Reduced Scale Tests of Pier Walls Under Cyclic Loading for Seismic Retrofit," Draft report to Caltrans, Division of Structures, November 1991.

Bent #	EQ Load Case	Max. Flexural Ductility Ratio Top	Max. Flexural Ductility Ratio Bottom	Shear* Demand to Capacity Ratio
2	1			
	2			
3	1			
	2			

Summary of Flexural and Shear Demand to Capacity Ratios

Figure 8

*a Shear demands are computed based on the lesser of elastic ARS shear and plastic shear values.

b Shear capacity is based on allowable values outside plastic hinge region (see Attachment B).

Note: The table above shows the minimum amount of information to be presented at a strategy meeting. Additional results may be provided if deemed necessary by the project engineer.

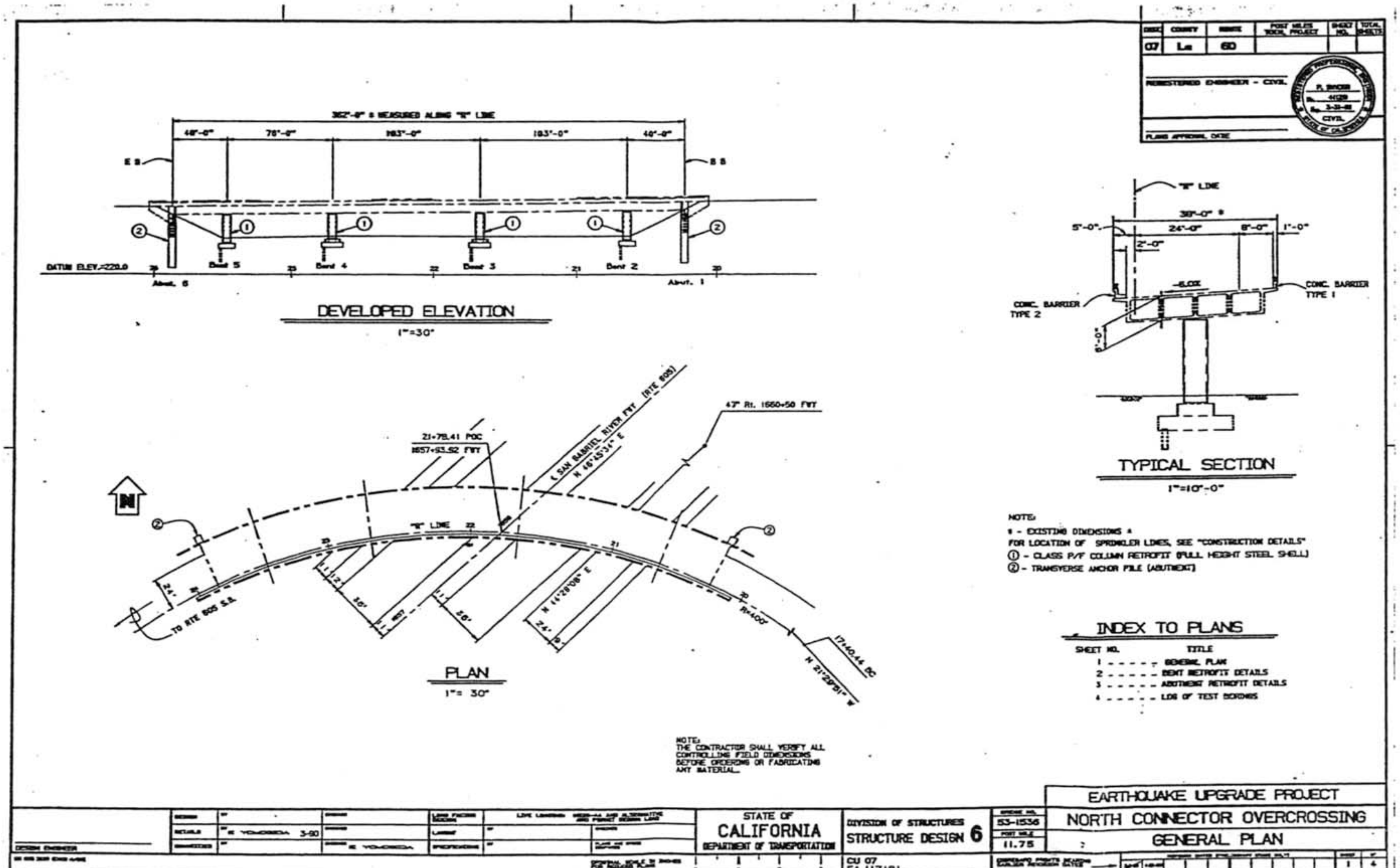

DEVELOPED ELEVATION
1"=30'
PLAN
1"= 30'
TYPICAL SECTION
1"=10'-0"
"E" LINE
TO RTE 605 S.B.
47' Rt. 1660+50 FWY
DATUM ELEV.=220.0
Abut. 6
Bent 5
Bent 4
Bent 3
Bent 2
Abut. 1
CONC. BARRIER TYPE 2
CONC. BARRIER TYPE 1
NOTE:
* - EXISTING DIMENSIONS *
FOR LOCATION OF SPRINKLER LINES, SEE "CONSTRUCTION DETAILS"
① - CLASS P/F COLUMN RETROFIT (FULL HEIGHT STEEL SHELL)
② - TRANSVERSE ANCHOR PILE (ABUTMENT)
INDEX TO PLANS
SHEET NO.
TITLE
1 - - - - - GENERAL PLAN
2 - - - - - BENT RETROFIT DETAILS
3 - - - - - ABUTMENT RETROFIT DETAILS
4 - - - - - LOG OF TEST BORINGS
NOTE:
THE CONTRACTOR SHALL VERIFY ALL CONTROLLING FIELD DIMENSIONS BEFORE ORDERING OR FABRICATING ANY MATERIAL.
STATE OF
CALIFORNIA
DEPARTMENT OF TRANSPORTATION
DIVISION OF STRUCTURES
STRUCTURE DESIGN 6
53-1536
11.75
CU 07
EA 117181
EARTHQUAKE UPGRADE PROJECT
NORTH CONNECTOR OVERCROSSING
GENERAL PLAN

Figure 9

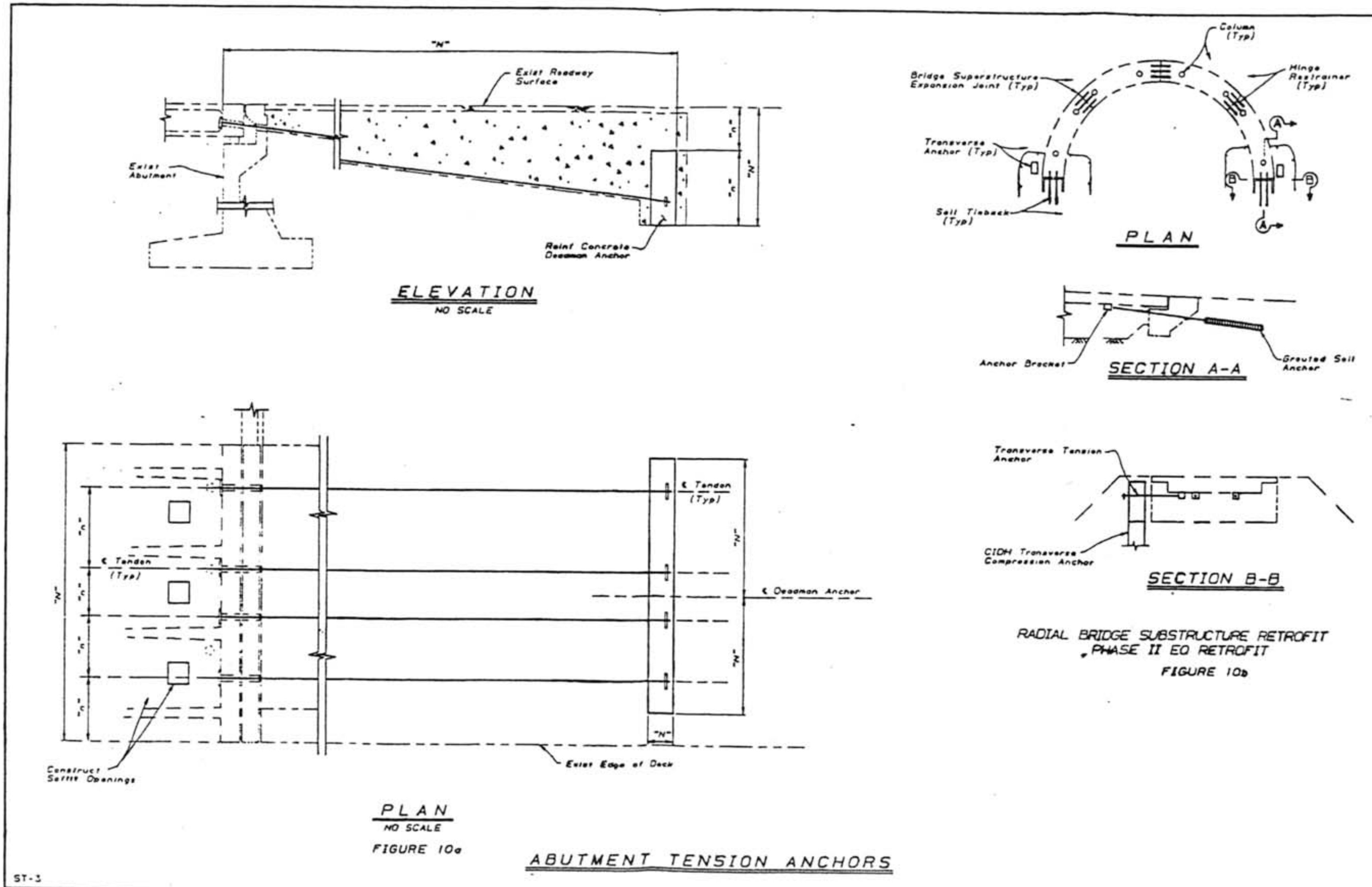

"N"
Exist Roadway Surface
Exist Abutment
Reinf Concrete Deadman Anchor
ELEVATION
NO SCALE
"n"
"N"
Construct Soffit Openings
℄ Tendon (Typ)
Exist Edge of Deck
℄ Deadman Anchor
PLAN
NO SCALE
FIGURE 10a
ABUTMENT TENSION ANCHORS
Column (Typ)
Bridge Superstructure Expansion Joint (Typ)
Hinge Restrainer (Typ)
Transverse Anchor (Typ)
Soil Tieback (Typ)
PLAN
Anchor Bracket
SECTION A-A
Grouted Soil Anchor
Transverse Tension Anchor
CIDH Transverse Compression Anchor
SECTION B-B
RADIAL BRIDGE SUBSTRUCTURE RETROFIT
PHASE II EQ RETROFIT
FIGURE 10b
ST-3

Figure 10

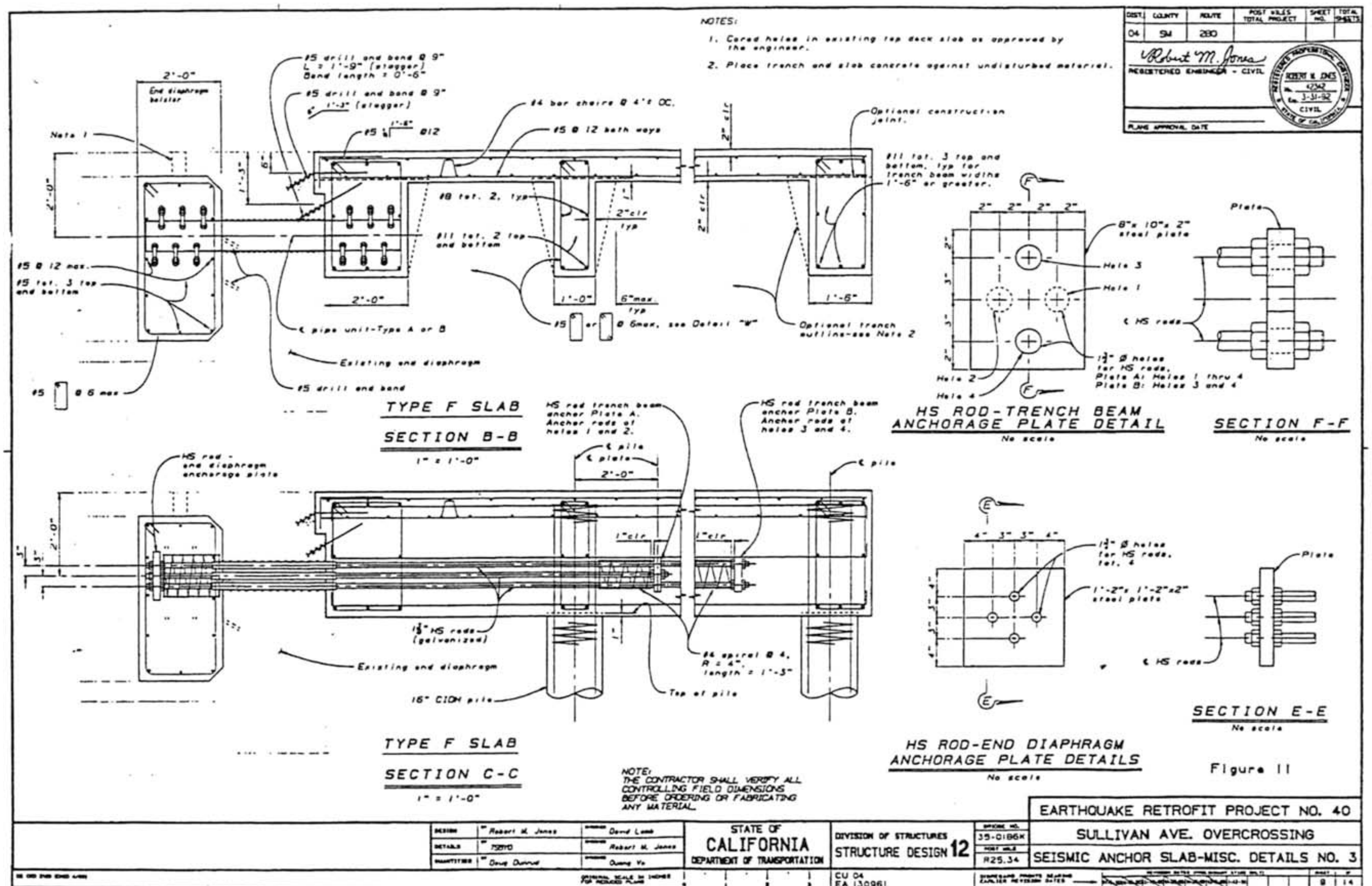

NOTES:
1. Cored holes in existing top deck slab as approved by the engineer.
2. Place trench and slab concrete against undisturbed material.
TYPE F SLAB
SECTION B-B
1" = 1'-0"
TYPE F SLAB
SECTION C-C
1" = 1'-0"
HS ROD-TRENCH BEAM ANCHORAGE PLATE DETAIL
No scale
SECTION F-F
No scale
HS ROD-END DIAPHRAGM ANCHORAGE PLATE DETAILS
No scale
SECTION E-E
No scale
Figure 11
NOTE: THE CONTRACTOR SHALL VERIFY ALL CONTROLLING FIELD DIMENSIONS BEFORE ORDERING OR FABRICATING ANY MATERIAL.
STATE OF CALIFORNIA
DEPARTMENT OF TRANSPORTATION
DIVISION OF STRUCTURES
STRUCTURE DESIGN 12
EARTHQUAKE RETROFIT PROJECT NO. 40
SULLIVAN AVE. OVERCROSSING
SEISMIC ANCHOR SLAB-MISC. DETAILS NO. 3
35-0186K
R25.34
CU 04
EA 130961
REGISTERED ENGINEER - CIVIL

Figure 11

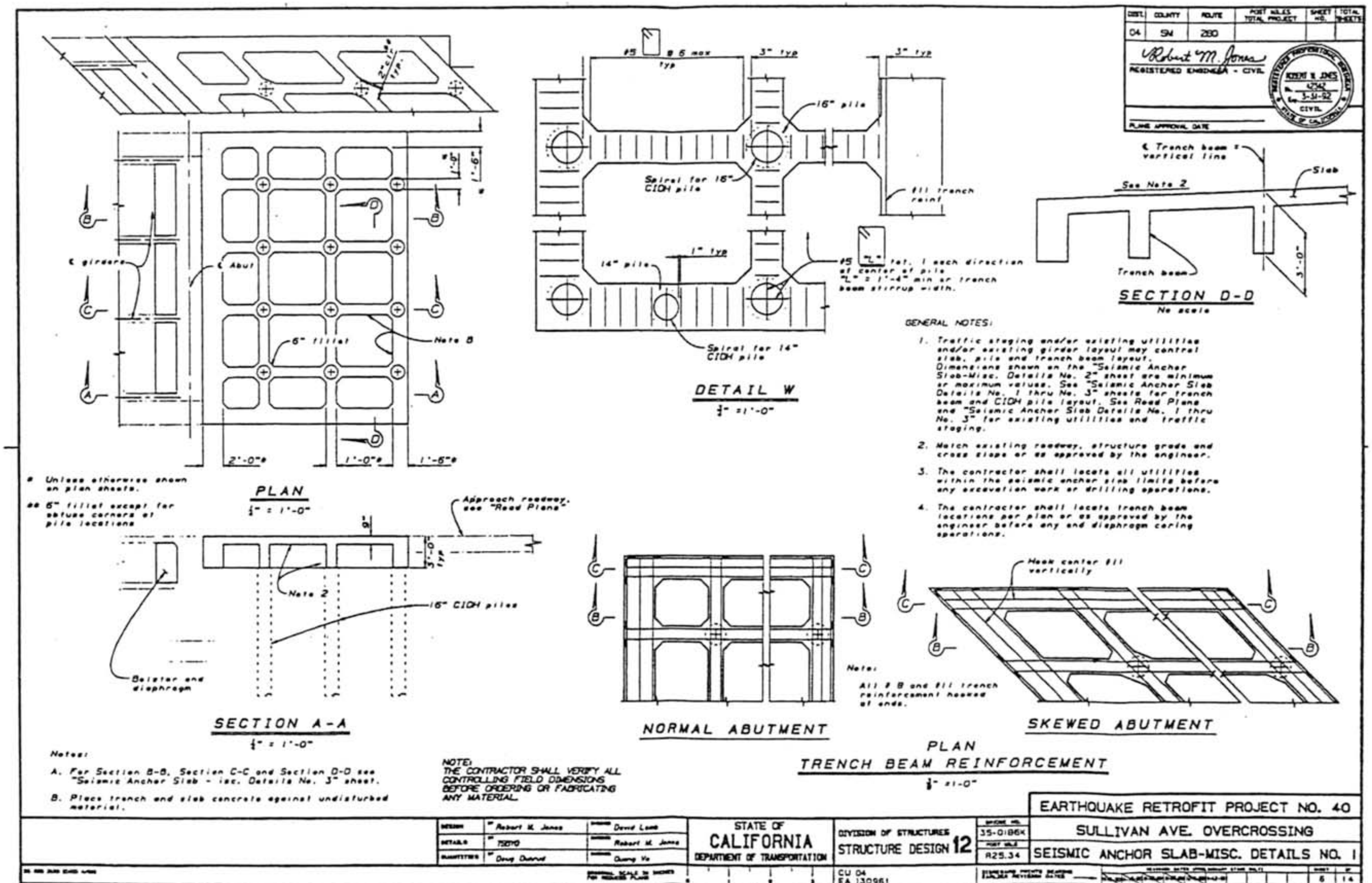

PLAN
½" = 1'-0"
SECTION A-A
½" = 1'-0"
DETAIL W
¾" = 1'-0"
SECTION D-D
No scale
NORMAL ABUTMENT
SKEWED ABUTMENT
PLAN
TRENCH BEAM REINFORCEMENT
¼" = 1'-0"
℄ girders
℄ Abut
6" fillet
Note B
Note 2
16" CIDH piles
Bolster and diaphragm
Approach roadway, see "Road Plans"
16" pile
14" pile
Spiral for 16" CIDH pile
Spiral for 14" CIDH pile
#5 @ 6 max typ
3" typ
1" typ
#11 trench reinf
#5 "L" tot. 1 each direction at center of pile "L" = 1'-4" min or trench beam stirrup width.
℄ Trench beam = vertical line
See Note 2
Slab
Trench beam
Hook center #11 vertically
Note: All # 8 and #11 trench reinforcement hooked at ends.
* Unless otherwise shown on plan sheets.
** 6" fillet except for obtuse corners at pile locations
2'-0"*
1'-0"*
1'-6"*
3'-0"
GENERAL NOTES:
1. Traffic staging and/or existing utilities and/or existing girder layout may control slab, pile and trench beam layout. Dimensions shown on the "Seismic Anchor Slab-Misc. Details No. 2" sheet are minimum or maximum values. See "Seismic Anchor Slab Details No. 1 thru No. 3" sheets for trench beam and CIDH pile layout. See Road Plans and "Seismic Anchor Slab Details No. 1 thru No. 3" for existing utilities and traffic staging.
2. Match existing roadway, structure grade and cross slope or as approved by the engineer.
3. The contractor shall locate all utilities within the seismic anchor slab limits before any excavation work or drilling operations.
4. The contractor shall locate trench beam locations per plan or as approved by the engineer before any end diaphragm coring operations.
Notes:
A. For Section B-B, Section C-C and Section D-D see "Seismic Anchor Slab - isc. Details No. 3" sheet.
B. Place trench and slab concrete against undisturbed material.
NOTE: THE CONTRACTOR SHALL VERIFY ALL CONTROLLING FIELD DIMENSIONS BEFORE ORDERING OR FABRICATING ANY MATERIAL.
REGISTERED ENGINEER - CIVIL
STATE OF CALIFORNIA
DEPARTMENT OF TRANSPORTATION
DIVISION OF STRUCTURES
STRUCTURE DESIGN 12
EARTHQUAKE RETROFIT PROJECT NO. 40
SULLIVAN AVE. OVERCROSSING
SEISMIC ANCHOR SLAB-MISC. DETAILS NO. 1
35-0186K
R25.34
CU 04
EA 130961
04
SM
280

Figure 12

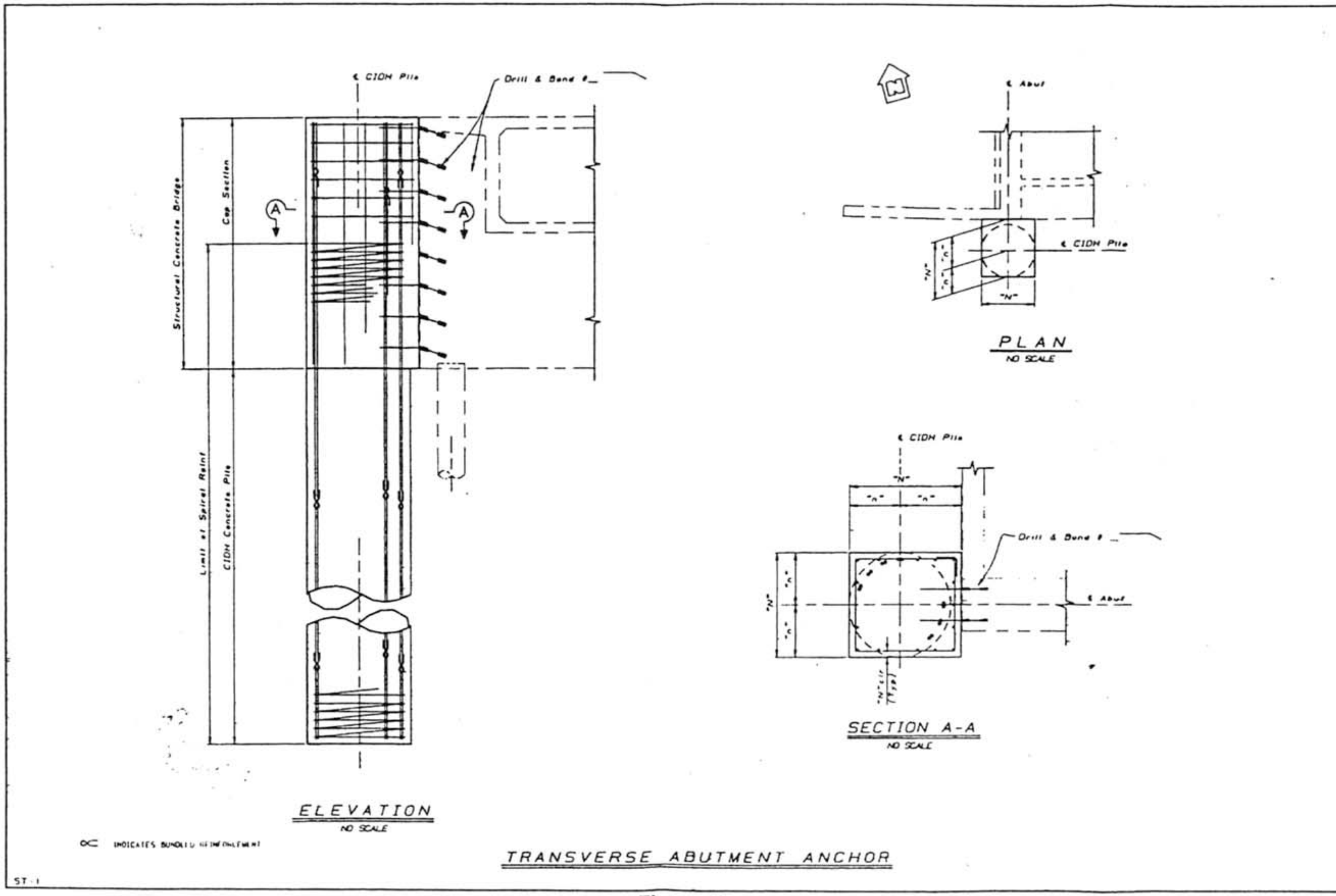

℄ CIDH Pile
Drill & Bond #_
Structural Concrete Bridge
Cap Section
Limit of Spiral Reinf
CIDH Concrete Pile
ELEVATION
NO SCALE
INDICATES BUNDLED REINFORCEMENT
℄ Abut
℄ CIDH Pile
PLAN
NO SCALE
SECTION A-A
NO SCALE
TRANSVERSE ABUTMENT ANCHOR
ST-1

Figure 13

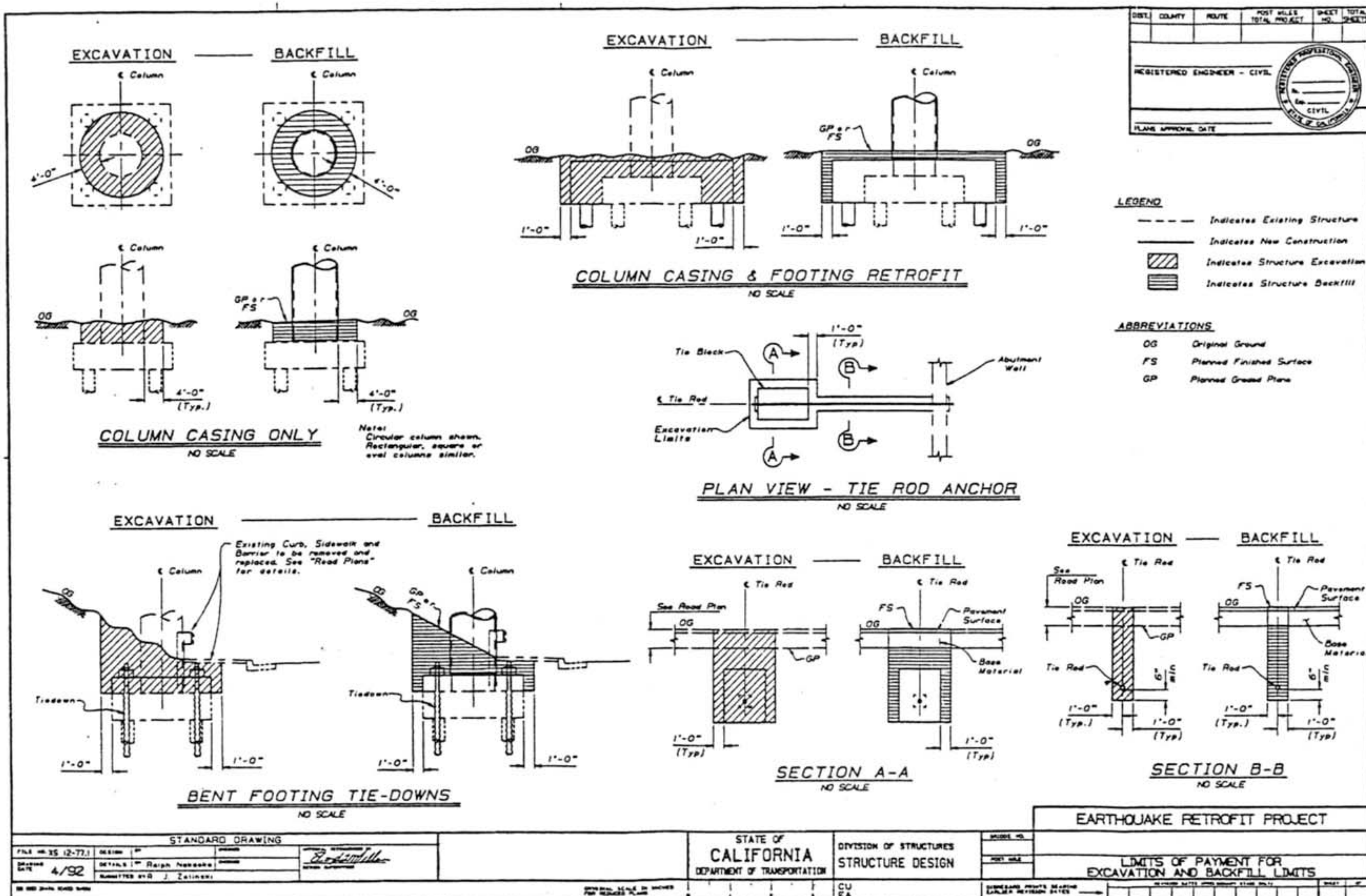

EXCAVATION — BACKFILL
COLUMN CASING ONLY
NO SCALE
Note: Circular column shown. Rectangular, square or oval columns similar.
COLUMN CASING & FOOTING RETROFIT
NO SCALE
PLAN VIEW - TIE ROD ANCHOR
NO SCALE
Tie Block
Tie Rod
Excavation Limits
Abutment Wall
BENT FOOTING TIE-DOWNS
NO SCALE
Existing Curb, Sidewalk and Barrier to be removed and replaced. See "Road Plans" for details.
Tiedown
SECTION A-A
NO SCALE
SECTION B-B
NO SCALE
Pavement Surface
Base Material
LEGEND
Indicates Existing Structure
Indicates New Construction
Indicates Structure Excavation
Indicates Structure Backfill
ABBREVIATIONS
OG Original Ground
FS Planned Finished Surface
GP Planned Graded Plane
REGISTERED ENGINEER - CIVIL
STANDARD DRAWING
STATE OF CALIFORNIA
DEPARTMENT OF TRANSPORTATION
DIVISION OF STRUCTURES
STRUCTURE DESIGN
EARTHQUAKE RETROFIT PROJECT
LIMITS OF PAYMENT FOR EXCAVATION AND BACKFILL LIMITS

Figure 14